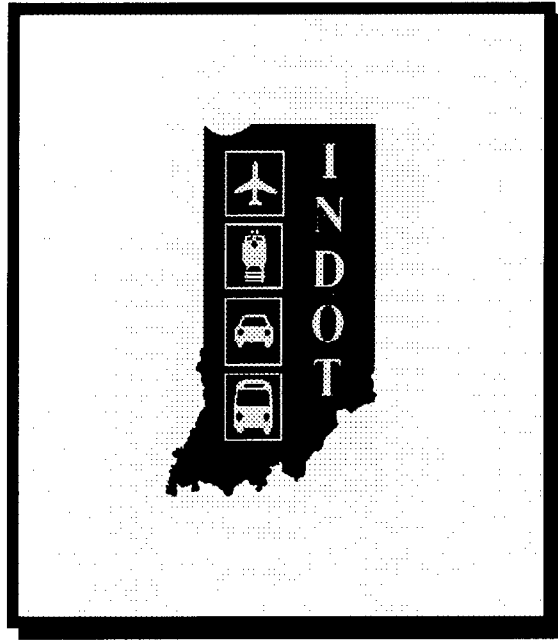# DIVISION OF RESEARCH

VERIFICATION OF DESIGN PARAMETERS

FOR OVERLAID RUBBLIZED

PCC PAVEMENT

Final Report

December 1997

Indiana Department of Transportation
Division of Research

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Revised Final Report Title: "Verification of Design Parameters for Overlaid Rubblized PCC Pavement," Final Report, December 1997.

Comments:

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VERIFICATION OF DESIGN PARAMETERS FOR OVERLAID RUBBLIZED PCC PAVEMENT

Introduction

Rubblizing is a relatively new technique for the rehabilitation of Portland Cement Concrete (PCC) pavements in which the concrete slab is reduced to an aggregate base material through the application of resonant frequency vibrations to the slab surface.

In 1991, the Indiana Department of Transportation (INDOT) awarded a construction contract to apply the technique of rubblizing to a portion of US-41 in Benton County in preparation for overlay. The Division of Research evaluated the pavement both prior to rubblizing and again after rubblizing and overlay in order to estimate the AASHTO "layer coefficient" of the rubblized concrete slabs. The layer coefficient parameter is required by the AASHTO pavement design procedure; however, there is currently little existing data upon which to make such an estimate. At the time of the US-41 contract award, only 19 pavement sections in 9 states had been rubblized. The layer coefficient for the rubblized PCC layer for most of these designs had been extrapolated from AASHTO recommendations for crushed aggregate base.

In 1991, the PCS/Law Company performed an analysis of the 19 existing sections and recommended layer coefficient values in the practical range of 0.23 to 0.31. This range is useful to the extent that it demonstrates that the layer coefficient of a rubblized concrete pavement is approximately twice that of a crushed aggregate base (layer coefficient = 0.14).

Findings

The results of this study allow the following conclusions to be made:

1. Rubblizing appears to provide a uniform, high-strength, granular base/subbase for the asphalt concrete overlay.

2. A layer coefficient of 0.22 is reasonable for rubblized PCC pavement. This value is approximately 1.5 that of a

crushed aggregate (0.14) and represents a conservative value.

3. Rubblization appears to be a valid practice for INDOT to use in the rehabilitation of PCC pavements.

Implementation

If INDOT continues to use the rubblization technique in pavement rehabilitation, a further study encompassing slabs of different thicknesses on different subgrades should be undertaken for confirmation of the values reported herein. If different rubblizing equipment is developed, this factor should also be examined.

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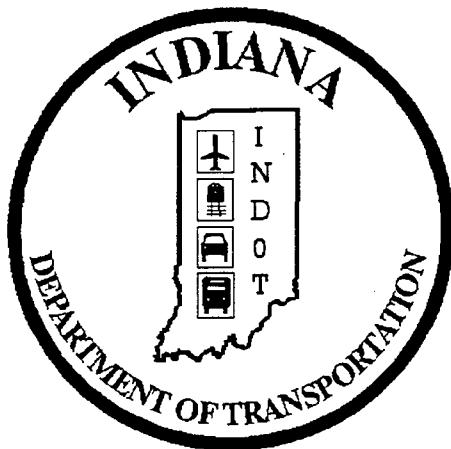
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FINAL REPORT

VERIFICATION OF DESIGN PARAMETERS
FOR
OVERLAID RUBBLIZED PCC PAVEMENT

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Abstract

The objective of rubblization is to eliminate reflection cracking in the HMA overlay by the total destruction of the existing slab action. Rubblization is applicable when there is little potential of retaining significant slab integrity and structural capacity of the original JRCP. It has also been used successfully for rehabilitation of other PCC pavement types. Typically, the slab is reduced into pieces less than 300mm in size. Subsequently, the slab is diminished to a high-strength granular base. Restoration of the structural capacity is accomplished in the overlay process by thickening the HMA overlay.

In 1991, INDOT awarded a contract to apply rubblization technique to a portion of US-41. The pavement sections were evaluated prior to and again after rubblizing and overlay to estimate the AASHTO layer coefficient of rubblized concrete pavements. The layer coefficient determined in this study (a = 0.25) represents a value two standard deviations less than that reported by PCS/Law. It should, therefore, represent a conservative value.

Currently, INDOT uses a layer coefficient of 0.20 for rubblized PCC pavements. Based on the results of this study, the layer coefficient can be set within two standard deviations of the mean (i.e. 0.22). This layer coefficient of 0.22 represents a conservative value that is recommended for rubblized PCC pavements with similar conditions. If INDOT continues to use the rubblization technique in pavement rehabilitation, a further study encompassing slabs of different thicknesses on different subgrades should be undertaken for confirmation of the values reported herein. If different rubblizing equipment is developed, this factor should also be examined.

ACKNOWLEDGMENT

IMPLEMENTATION SUGGESTIONS

The Results of this study allows the following implementation suggestions:

- Rubblizing appears to provide a uniform, high-strength, granular base/subbase for the asphalt concrete overlay.

- To ensure structural adequacy, no recommendation is made to increase the layer coefficient from 0.20 to 0.25. However, a layer coefficient of 0.22 appears to be reasonable for rubblized PCC pavement with similar conditions. This layer coefficient is set within two standard deviation of the mean. In addition, this value is approximately 1.5 that of a crushed aggregate (0.14) and therefore represents a conservative value.

- Rubblization appears to be a valid practice for INDOT to use in the rehabilitation of PCC pavements.

- If INDOT continues to use rubblization as a technique in the rehabilitation of PCC pavements, further studies are recommended to verify values reported in this study. Additional studies should encompass slabs of different thicknesses on different subgrades. If new equipment is employed, this factor should also be examined.

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1.0 INTRODUCTION

Rubblizing is a relatively new technique for the rehabilitation of Portland Cement Concrete (PCC) pavements in which the concrete slab is reduced to an aggregate base material through the application of resonant frequency vibrations to the slab surface. Prior to the development of rubblization, four PCC pavement rehabilitation techniques were employed. These are:

    1. Overlay,

    2. Crack-and seat with overlay,

    3. Break-and-seat with overlay, and

    4. Total reconstruction.

With the exception of the reconstruction option, which is the most expensive in terms of initial cost, these options have exhibited various shortcomings in either field operations or final performance.

In 1991, the Indiana Department of Transportation (INDOT) awarded a construction contract to apply the technique of rubblizing to a portion of US-41 in Benton County in preparation for overlay. The Division of Research evaluated the pavement both prior to rubblizing and again after rubblizing and overlay in order to estimate the AASHTO "layer coefficient" of the rubblized concrete slabs. The layer coefficient parameter is required by the AASHTO pavement design procedure; however, there is currently little existing data upon which to make such an estimate. At the time of the US-41 contract award, only 19 pavement sections in 9 states had been rubblized. The

layer coefficient for the rubblized PCC layer for most of these designs had been extrapolated from AASHTO recommendations for crushed aggregate base.

In a report dated June 1991, the PCS/Law Company (*1*) performed an analysis of the 19 existing sections and recommended layer coefficient values in the practical range of 0.23 to 0.31. This range is useful to the extent that it demonstrates that the layer coefficient of a rubblized concrete pavement is approximately twice that of a crushed aggregate base (layer coefficient = 0.14).

The present study is limited to an analysis of the relative strengths of the US-41 pavement prior to, and after rubblizing. The study includes a pre-analysis of the existing pavement to ensure section uniformity, and a post-construction analysis designed to yield estimates for the layer coefficient of the in-situ rubblized pavement.

2.0 METHOD OF COMPARISONS

2.1 Crack and Seat

The objective of crack and seat is to eliminate reflection cracking in the Hot Mix Asphalt (HMA) overlay. This is accomplished by creating concrete slabs that are small enough to reduce horizontal slab movement such that thermal stresses in the HMA overlay can be accommodated. Ideally, when crack and seat is applied, slabs should be from 0.3-1.0 meters (1-3 feet) in length. Cracking is intended to produce tight cracks that permit load transfer with little loss of structural value. Seating of the cracked slabs is intended to re-establish support between the base or subbase and the PCC slab. Crack and seat has been used successfully for rehabilitation of Jointed Plain Concrete Pavements (JPCP)

that are generally structurally sound, but have functional distresses in the form of roughness, patching , spalling, etc.

2.2 Break and Seat

The objective of break and seat is similar to that of crack-and seat. However, break and seat is used with Jointed Reinforced Concrete Pavements (JRCP) and requires that the bond between the concrete and reinforcement steel be destroyed. This lack of bond minimally reduces the differential movements at working joints and cracks (*1*). Because of the bond disruption, the amount of energy required to reduce the slab size is greater than that required for crack and seat. The reduction of structural capacity due to the disruption of the reinforcing steel/concrete bond should be considered during overlay design thickness. If pavement deterioration is such that little slab integrity can be preserved after breaking, rubblization or reconstruction should be considered as alternative procedures.

2.3 Rubblize and Overlay

The objective of rubblization is to eliminate reflection cracking in the HMA overlay by the total destruction of the existing slab action. Rubblization is applicable when there is little potential of retaining significant slab integrity and structural capacity of the original JRCP. It has also been used successfully for rehabilitation of other PCC pavement types. Typically, the slab is reduced into pieces less than approximately 300 mm (12 inches) in size. Subsequently, the slab is diminished to "a high-strength granular base (*1*)". Restoration of the structural capacity must be accomplished in the overlay process by thickening the HMA overlay.

3.0 SITE DESCRIPTION

The construction contract of this project covered a portion of US-41 in Benton County, extending from the northern limits of Boswell approximately 22 km (14 miles) north to the Benton/Newton County line. Most of the north-bound lanes were to be rubblized, with the exception of sections adjacent to various structures (bridges, culverts etc.). Only a limited section of the south-bound lanes were rubblized, approximately 14 km (9 miles). The rehabilitation work was divided into two contracts. The northern most contract contained approximately 9 km (6 miles) of the north-bound lanes and 14 km (9 miles) of the south-bound lanes. The southern most contract contained approximately 14 km (9 miles) of the north-bound lanes only. The northern contract was contained between US-24 and the US-41/52 intersection. The southern contract was between SR-352 and the US-41/52 intersection.

The work was divided into sections some of which were rubblized; the remainder were patched and overlaid with asphalt concrete. Within the limits of the rubblizing contract, the north-bound pavement consisted of 13 km (8 miles) of 200 mm (8 inches) thick CRCP on 150 mm (6 inches) of uniform subbase, and 9 km (5.7 miles) of 255 mm (9 inches) thick RCP slabs on a variable 125-200 mm (5-8 inches) thick subbase. The south-bound pavement consisted of 14 km (9 miles) of 255 mm (9 inches) thick RCP slabs on a variable 125-200 mm (5-8 inches) thick subbase. Table 1 illustrates the construction activities on these pavement sections.

TABLE 1 Construction Activities.

Location	Activity	Length (km)
South-Bound (N)	Rubblizing	13.65
	Patch & Overlay	0.78
	Bridge	0.14
	TOTAL	14.57
North-Bound (S)	Rubblizing	9.00
	Patch & Overlay	3.63
	Remove & Replace	0.06
	Bridge	0.05
	TOTAL	12.75
North-Bound (N)	Rubblizing	7.36
	Patch & Overlay	1.78
	Bridge	0.14
	TOTAL	9.12

The existing concrete pavement was generally in poor to very poor condition. Most joints in JRCP exhibited significant faulting and major damage and many slabs were cracked; punchouts were common in CRCP. There was visual evidence of pumping. Much of pavement had been undersealed with an asphalt compound. The District reports that more than one application of undersealing had been undertaken on this stretch. Truck traffic is generally heavy (approximately 3.9 million ESALs), providing a convenient route south from Chicago to Evansville, Indiana with connections to both I-70 and I-74.

4.0 PRE-CONSTRUCTION TESTING

In preparation for the rubblizing contract, the Division of Research tested the entire length of both north-bound and south-bound lanes using a Dynatest Falling Weight Deflectometer (FWD). This information was used to select two test sections, each 1.6 km (1 mile) in length. The sections identified were (stations are based on the mile/foot):

Section 1: From station 409+00 to 460+00 (north-bound driving lane) between Boswell and the US-52 junction.

Section 2: From station 515+00 to 525+00 and from station 210+00 to 250+00 (continuous, with a station equation at 525+00), north of the US-52 junction.

5.0 PRE-CONSTRUCTION ANALYSIS

In performing the analysis to determine the moduli of the concrete and subgrade, it was necessary to employ an algorithm appropriate to the construction. There are many software solutions available, such as MODULUS, ELSBACK, WESBACK, MODCOMP, etc. However, while these are generally applicable to multi-layer elastic systems, they are better suited to the analysis of flexible pavement systems. K.T. Hall, et al. (4) report an improved system for the analysis of rigid pavement systems that has since been incorporated in the AASHTO (2) recommendations for pavement analysis of rigid pavement systems. This improved analysis was applied to the data from this project.

The recorded pavement deflection basins are reduced to a normalized parameter, AREA2, by dividing the area of the deflection basin between the axis of load and a radial offset of 914 mm (36 inches) by the deflection at 0 mm; only the deflections measured at 0, 305, 610 and 914 mm (0, 12, 24 and 36 inches) are used.
Thus:

$$AREA2 = 6 \left(\frac{d(0) + 2d(12) + 2d(24) + d(36)}{d(0)} \right) \qquad (1)$$

This parameter has been analytically identified (3) as having a unique relationship to the radius of relative stiffness (l_k or l_e) of the pavement system. The Relative stiffness is calculated as:

$$l_k = \left(\frac{E_{pcc} D_{pcc}^3}{12(1 - \mu^2_{pcc})k} \right)^{1/4}$$

$$l_e = \left(\frac{E_{pcc} D_{pcc}^3 \left(1 - \mu^2_s\right)}{6 \left(1 - \mu^2_{pcc}\right) E_s} \right)^{1/3} \qquad (2)$$

where,

E_{pcc} = the elastic modulus of the PCC slab,

E_s = elastic modulus of the supporting medium,

μ_{pcc} = the Poisson's ratio of the concrete slab,

μ_s = the Poisson's ratio of the supporting medium,

D_{pcc} = the thickness of the slab (inches), and

k = the modulus of the subgrade reaction.

The difference between these two parameters relates to computing subgrade support models, the dense liquid model (l_k) and the elastic solid model (l_e). Traditional rigid pavement design is based on the dense liquid model. However, highway designers (as opposed to airfield designers) rarely measure the modulus of subgrade reaction, k, taking instead an estimate based on correlation with the California Bearing Ratio (CBR). Since the advent of FWD backcalculation technology, the use of the elastic solid model has become more applicable, because estimates of the in-situ elastic response of the subgrade may be relatively easily obtained. The elastic solid model was used for the US-41 analysis. Hall (4) provides a relationship between the radius of relative stiffness, l_e, and AREA2.

$$l_e = \left(\frac{\ln\left(\frac{36 - AREA2}{4521.676} \right)}{-3.654} \right)^{\frac{1}{0.187}} \tag{3}$$

Using this estimate of the modulus of subgrade reaction, l_e, the elastic modulus of the subgrade, E_s, may be estimated using Losberg's deflection equation.

$$E_s = \left[\frac{2p(1 - \mu_s^2)}{d_0 l_e} \right] \left[.019245 + .0272\left(\frac{a}{l_e} \right) + 0.0199\left(\frac{a}{l_e} \right)^2 \ln\left(\frac{a}{l_e} \right) \right] \tag{4}$$

Applying this result to Equation 2, an estimate of the elastic modulus of the PCC slab may be obtained.

$$E_s = \frac{E_c D_c{}^3 (1 - \mu_s^2)}{6 l_e{}^3 (1 - \mu_s^2)}$$ (5)

When these transformations are applied to the FWD data obtained from Sections 1 and 2 on US-41, the overall uniformity of the sections is confirmed. The results are shown in Table 2. From these results, it is clear that the elastic moduli of the concrete slabs tested are not significantly different, and do not differ significantly from the normal design value of 27,579 MPa (4,000,000 psi). Since the two sections are physically removed from each other by more than 1.6 km (1 mile), the difference in the subgrade moduli (which includes the subbase layer) is within normal soil variability expectations.

TABLE 2 Pre-Rubblization Analysis Summary.

	Concrete Modulus (MPa)		Subgrade Modulus (MPa)	
	Mean	Std. Dev.	Mean	Std. Dev.
Section 1	26,794	5,395	159	23
Section 2	25,478	3,329	195	31

6.0 POST-RUBBLIZING ANALYSIS

After the patching, rubblizing, and overlay procedures were completed, FWD tests were repeated at the same locations as before. In this instance, since the pavement now comprises a flexible structure, the previous analysis was no longer appropriate. Two analyses were therefore undertaken. The first, using BOUSDEF software (Oregon State University), was used solely to determine the extent, if any, of subgrade nonlinear elastic

response. BOUSDEF is the only program available to the Division of Research that can make this determination. It is known that many subgrade soils exhibit a non-linear elastic response that is different for cohesive and non-cohesive soils. Thus,

$$E = K_1 \Theta^{k_2}$$

$$E = K_3 \sigma_d^{k_4} \qquad (6)$$

where,

Θ = the first stress invariant,

σ_d = the deviator stress, and

k_i = experimental constants.

The BOUSDEF analysis revealed no consistent pattern (-0.2 < $k_{2,4}$ < 0.32), and the linear option ($k_{2,4}$ = 0) fell well within one standard deviation of the mean. Consequently, linear elastic response was accepted for the subgrade.

Having determined that the subgrade was essentially linear elastic, the same data was analyzed using MODULUS. This program is capable of backcalculating layer moduli and detecting the presence of any real, or effective, lower rigid boundary, which can seriously modify the response of a pavement system to FWD loading. With the exception of less than half a dozen instances, all observations were reported to have a depth to rigid layer in excess of 7.5 m (25 feet). Consequently, for all practical purposes, an infinite bottom condition prevailed.

The average results of the MODULUS backcalculations are shown in Table 3. Once again there is a reasonable degree of uniformity. The difference in the modulus of the AC overlays in Sections 1 and 2 may be partially accounted for by the difference in

pavement temperatures. The two sections were tested on successive days, and a 7°C (12°F) difference was measured in-situ. The difference between the subgrade moduli computed in the two sections may be accounted for using the same logic as before. The moduli of the rubblized layers in the two sections are statistically identical.

TABLE 3 Summary of Moduli from Rubblized Pavements.

	AC Modulus (MPa)		Rubblized layer (MPa)		Subgrade Modulus (MPa)	
	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
Section 1	8190	1101	1253	376	128	20
Section 2	5967	807	1152	195	159	26

It should be noted that the reported subgrade moduli differ between the before and after analyses. This may be a reflection of the different modes by which rigid and flexible pavements transmit load to the subgrade. Rigid pavements "bridge" the subgrade and transfer the load in an entirely different manner from flexible pavements.

7.0 LAYER COEFFICIENTS

The concept of the layer coefficients is derived from the AASHO Road Test in which the structural geometry is reduced to a single parameter structural number (SN = a_1t_1 + a_2t_2 + a_3t_3). This reduction is based upon the development of measured levels of specific distresses (cracking, patching, roughness and rutting). The linear concept of structural number has been misapplied to structural strength by analogy to the Odemark hypothesis (5) of equivalent thickness wherein

$$h_e = h_1 \sqrt[3]{\frac{E_1}{E_{ref}}} + h_2 \sqrt[3]{\frac{E_2}{E_{ref}}} + h_3 \sqrt[3]{\frac{E_3}{E_{ref}}} \qquad (7)$$

where,

h_e = the equivalent thickness,

h_1, h_2, h_3 = the thickness of layer 1, 2, and 3 respectively,

E_{ref} = the reference modulus (75,644 MPa), and

E_1, E_2, E_3 = the elastic modulus of 1st, 2nd and 3rd layers respectively.

By analogy, the layer coefficient of the second layer, a_2, would be evaluated as

(1)

$$a_2 = \sqrt[3]{\frac{E_1}{E_{ref}}} = 0.0045 \sqrt[3]{E_2} \qquad (8)$$

where, in order to provide results equivalent to the layer coefficients derived at the AASHO Road Test, E_{ref} = 75,664 MPa (10,974,000 psi). These relationships are artificial and applied in a manner incompatible with the derivation of layer coefficients at the AASHO Road Test. Nonetheless, the academic and research fraternity have adopted this method as an empirical means of estimating actual layer coefficients.

Equation 8 has been applied to the data from Sections 1 and 2 and the results are reported in Tables 4 and 5.

US-41: Rubblized Section #1

| Station | Deflection (mils) | | Moduli (ksi) | | | |
	D(0)	D(7)	E1	E2	E3	a2
409	5.20	2.19	1179	271.0	14.6	0.29
410	5.34	2.17	1142	222.9	14.9	0.27
411	5.08	2.17	1536	171.4	15.4	0.25
412	5.22	1.91	1176	124.1	17.6	0.22
413	5.21	1.93	1237	119.1	17.3	0.22
414	5.64	2.02	1056	110.9	16.8	0.22
415	4.58	1.71	1284	183.0	19.5	0.26
416	4.63	1.73	1310	168.7	19.3	0.25
417	4.94	1.77	1087	168.5	19.0	0.25
418	5.02	2.00	1175	229.1	16.3	0.28
419	4.86	1.98	1375	210.6	16.4	0.27
420	4.82	2.02	1458	219.8	16.1	0.27
421	4.86	1.74	948	240.1	19.1	0.28
422	4.46	1.68	1356	185.7	19.8	0.26
423	4.57	1.71	1394	151.1	19.6	0.24
424	4.73	1.80	1009	302.1	18.0	0.30
425	4.55	1.79	1249	257.9	18.1	0.29
426	4.69	1.91	1190	302.0	16.8	0.30
427	6.47	2.35	921	105.2	14.2	0.21
428	4.98	2.03	1311	209.6	16.1	0.27
429	5.35	2.19	1082	256.4	14.6	0.29
430	5.47	2.11	992	213.1	15.5	0.27
431	4.25	1.51	1412	135.8	22.6	0.23
432	4.52	1.57	1169	162.0	21.6	0.25
433	4.74	1.59	1039	158.8	21.3	0.24
434	5.75	2.46	1361	152.7	13.6	0.24
435	4.74	1.86	1041	327.3	17.2	0.31
436	4.66	1.84	1347	211.0	17.7	0.27
437	4.78	1.82	1230	198.8	18.0	0.26
438	4.63	1.64	1229	147.2	20.6	0.24
439	4.85	1.56	1019	123.9	21.9	0.22
440	4.75	1.81	1268	190.5	18.2	0.26
441	4.76	1.47	883	165.4	23.3	0.25
442	4.63	1.45	937	162.5	23.6	0.25
443	4.44	1.45	1100	150.3	23.6	0.24
444	4.07	1.35	1066	247.2	24.9	0.28
445	3.98	1.34	1243	192.1	25.3	0.26
446	4.06	1.35	1169	199.4	25.0	0.26
447	5.04	1.91	1315	134.2	17.4	0.23
448	5.64	1.84	899	107.5	18.5	0.21
449	5.65	1.87	925	110.6	18.0	0.22
450	4.75	1.74	1248	156.5	19.3	0.24
451	4.80	1.90	1321	206.7	17.2	0.27
452	4.79	1.81	1382	139.3	18.5	0.23
453	5.48	1.91	1079	98.6	17.7	0.21
454	5.29	1.93	1239	95.0	17.5	0.21
455	4.69	1.63	1091	173.5	20.7	0.25
456	4.74	1.82	1182	232.1	17.9	0.28
457	4.59	1.82	1376	216.9	17.9	0.27
458	5.20	2.03	1267	160.6	16.3	0.24
459	5.15	2.04	1424	128.6	16.4	0.23
460	5.63	2.08	1039	144.8	16.0	0.24

Mean			1187.8	181.8	18.5	0.25
S.Dev			159.7	54.6	2.9	0.03
CoV			13.4	30.1	15.4	10.0

Table 4: US41: Rubblized Section #1

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US-41: Rubblized Section #2

Station	Deflection (mils)		Moduli (ksi)			
	D(0)	D(7)	E1	E2	E3	a2
515	5.02	1.84	1116	174.4	18.0	0.25
516	4.92	1.79	1177	154.6	18.7	0.24
517	5.34	1.64	796	142.3	20.7	0.23
518	5.14	1.77	900	196.3	18.9	0.26
519	4.81	1.64	1084	146.8	20.6	0.24
520	4.71	1.37	850	143.5	25.3	0.24
521	4.90	1.42	773	157.2	24.4	0.24
522	4.81	1.46	874	148.7	23.6	0.24
523	3.92	1.07	910	178.2	32.6	0.25
524	4.22	1.12	767	192.0	31.2	0.26
525	3.88	1.11	1021	165.9	31.3	0.25
210	4.66	1.35	830	158.8	25.6	0.24
211	4.72	1.39	801	172.5	25.0	0.25
212	4.67	1.41	863	167.8	24.4	0.25
213	4.72	1.43	944	124.3	24.3	0.22
214	4.51	1.35	931	150.5	25.5	0.24
215	4.55	1.39	985	135.0	24.8	0.23
216	4.50	1.40	989	150.7	24.5	0.24
217	4.29	1.40	1038	203.6	24.2	0.26
218	4.93	1.53	826	177.3	22.3	0.25
219	5.17	1.59	720	201.7	21.4	0.26
220	5.05	1.60	828	176.0	21.3	0.25
221	4.61	1.47	843	235.3	23.1	0.23
222	4.88	1.55	891	162.7	22.1	0.25
223	5.15	1.46	707	148.7	23.9	0.24
224	5.09	1.47	762	138.0	23.7	0.23
225	4.52	1.55	1003	231.3	21.5	0.26
226	4.52	1.54	1048	197.8	21.9	0.26
227	4.91	1.52	820	177.2	22.6	0.25
228	5.51	1.70	730	158.6	20.1	0.24
229	5.43	1.71	805	139.4	20.1	0.23
230	5.22	1.69	925	125.9	20.3	0.23
231	5.42	1.74	812	153.3	19.6	0.24
232	5.25	1.74	896	177.9	18.7	0.25
233	5.39	1.84	870	177.3	18.2	0.25
234	4.98	1.59	948	130.2	21.6	0.23
235	5.62	1.64	686	133.2	21.1	0.23
236	5.11	1.65	972	117.7	20.8	0.22
237	5.05	1.32	646	144.6	26.7	0.24
238	4.63	1.39	896	149.1	25.0	0.24
239	4.91	1.42	765	157.8	24.4	0.24
240	4.92	1.54	831	177.9	22.3	0.25
241	5.10	1.53	762	164.7	22.5	0.25
242	4.82	1.56	969	151.6	21.8	0.24
243	4.97	1.57	846	174.7	21.8	0.25
244	5.34	1.63	698	185.3	21.1	0.26
245	4.84	1.60	931	182.2	21.1	0.26
246	4.79	1.57	918	192.2	21.6	0.26
247	5.73	1.89	841	127.7	18.0	0.23
248	5.61	1.81	682	208.0	18.7	0.27
249	4.03	1.04	720	233.3	33.8	0.23
250	4.01	1.05	759	218.4	33.6	0.27

	Mean		865.5	167.1	23.1	0.25
	S.Dev		117.1	28.3	3.8	0.01
	CoV		13.5	17.0	16.3	5.6

Table 5: US41: Rubblized Section #2

Table 6 provides the summary of the derived layer coefficients for the rubblized layers from the two sections.

TABLE 6 Layer Coefficient Summary.

	Layer Coefficient	
	Mean	Std. Dev.
Section 1	0.25	0.03
Section 2	0.25	0.01

These results are remarkably consistent, and leave little doubt as to the rubblized layer coefficient mean value (a_2 = 0.25). The distributions of the derived layer coefficients do however differ, that of Section 2 being somewhat less variable than that observed in Section 1. However, the PCS/Law (*1*) report gives a mean value of a_2 = 0.33, σ = 0.15, which distribution is not the same as reported above, more notably in terms of the variability. The US-41 results reported above (μ = 0.25) correspond to a reliability of 94 percent (i.e. only 6 percent of the PCS/Law values are less than 0.25).

It should be borne in mind that the PCS/Law recommendations are based upon results from a number of states, and therefore reflect differences in material specifications and construction practices. They must therefore be expected to demonstrate a higher degree of variability than values obtained from a single state agency, and more specifically from a single stretch of highway. This is borne out by the difference in the observed variabilities (overall COV = 8 percent) which are significantly less than those reported by PCS/Law (COV = 13 percent). That the PCS/Law report

provides a greater mean value (a = 0.33) than that found in this project (a = 0.25) cannot be accounted for except by inference.

The apparent difference in average layer coefficient may be attributed to a number of causes. This includes different concrete strength specifications, different disruptive responses to the rubblizing equipment (i.e., the size distribution of the resulting rubblized concrete), and differences in rubblizing equipment (frequency of applied vibration, speed of travel, etc.).

8.0 DISCUSSION

Rubblizing concrete pavement reduces heavily distressed PCC slabs to an essentially uniform granular structure, thereby providing a structurally valid base/subbase layer in a flexible pavement structure. Rubblizing destroys the slab action of rigid pavements and this loss of structure must be accounted for in the overlay design thickness. In Indiana, an AASHTO Layer Coefficient of 0.25 appears to be reasonable. No adjustment should be made to this value for reliability. Reliability is dealt with separately in the AASHTO design process. The layer coefficient determined in this study (a = 0.25) represents a value two standard deviations less than that reported by PCS/Law. It should, therefore, represent a conservative value.

Currently, INDOT uses a layer coefficient of 0.20 for rubblized PCC pavements. Based on the results of this study, the layer coefficient can be set within two standard deviations of the mean (i.e. 0.22). This layer coefficient of 0.22 represents a conservative value that is recommended for rubblized PCC pavements with similar conditions.

Cores taken from the rubblized sections after overlay indicate that the thickness of the rubblized concrete slab is, within normal variability ranges, the same as the thickness of the unrubblized slabs at least for design purposes. If INDOT continues to use the rubblization technique in pavement rehabilitation, a further study encompassing slabs of different thicknesses on different subgrades should be undertaken for confirmation of the values reported herein. If different rubblizing equipment is developed, this factor should also be examined.

9.0 CONCLUSIONS AND RECOMMENDATIONS

The results of this study allow the following conclusions to be made:

1. Rubblizing appears to provide a uniform, high-strength, granular base/subbase for the asphalt concrete overlay.

2. To ensure structural adequacy, no recommendation is made to increase the layer coefficient from 0.20 to 0.25. However, a layer coefficient of 0.22 appears to be reasonable for rubblized PCC pavement with similar conditions. This layer coefficient is set within two standard deviation of the mean. In addition, this value is approximately 1.5 that of a crushed aggregate (0.14) and therefore represents a conservative value.

3. Rubblization appears to be a valid practice for INDOT to use in the rehabilitation of PCC pavements.

4. If INDOT continues to use rubblization as a technique in the rehabilitation of PCC pavements, further studies are recommended.

Additional studies should encompass slabs of different thicknesses on different subgrades. If new equipment is employed, this factor should also be examined.

10.0 REFERENCES

1. "Guidelines and Methodologies for the Rehabilitation of Rigid Highway Pavements using Asphalt Concrete Overlays", PCS/Law, June 1991.

2. "Guide for Design of Pavement Structures", AASHTO 1991.

3. Ioannides, A.M., "Dimensional Analysis in NDT Rigid Pavement Evaluation", Transportation Engineering Journal, ASCE, Vol. 116, No t. TE1, 1990.

4. Hall, K.T. and Mohseni, A., "Backcalculation of Asphalt Concrete Overlaid Portland Cement Concrete Pavement Layer Moduli", TRB Record 1293, 1991.

5. Odemark, N., "Undersokning av elasticitetsegenskaperna hos olika jodarter santteori for berakning av belaggninger enligt elasticitetsteorin", Statens Vaginstitut meddelande 77, 1949 (in Danish).

Figure 1. US41: Sect 1 - Subgrade Modulus
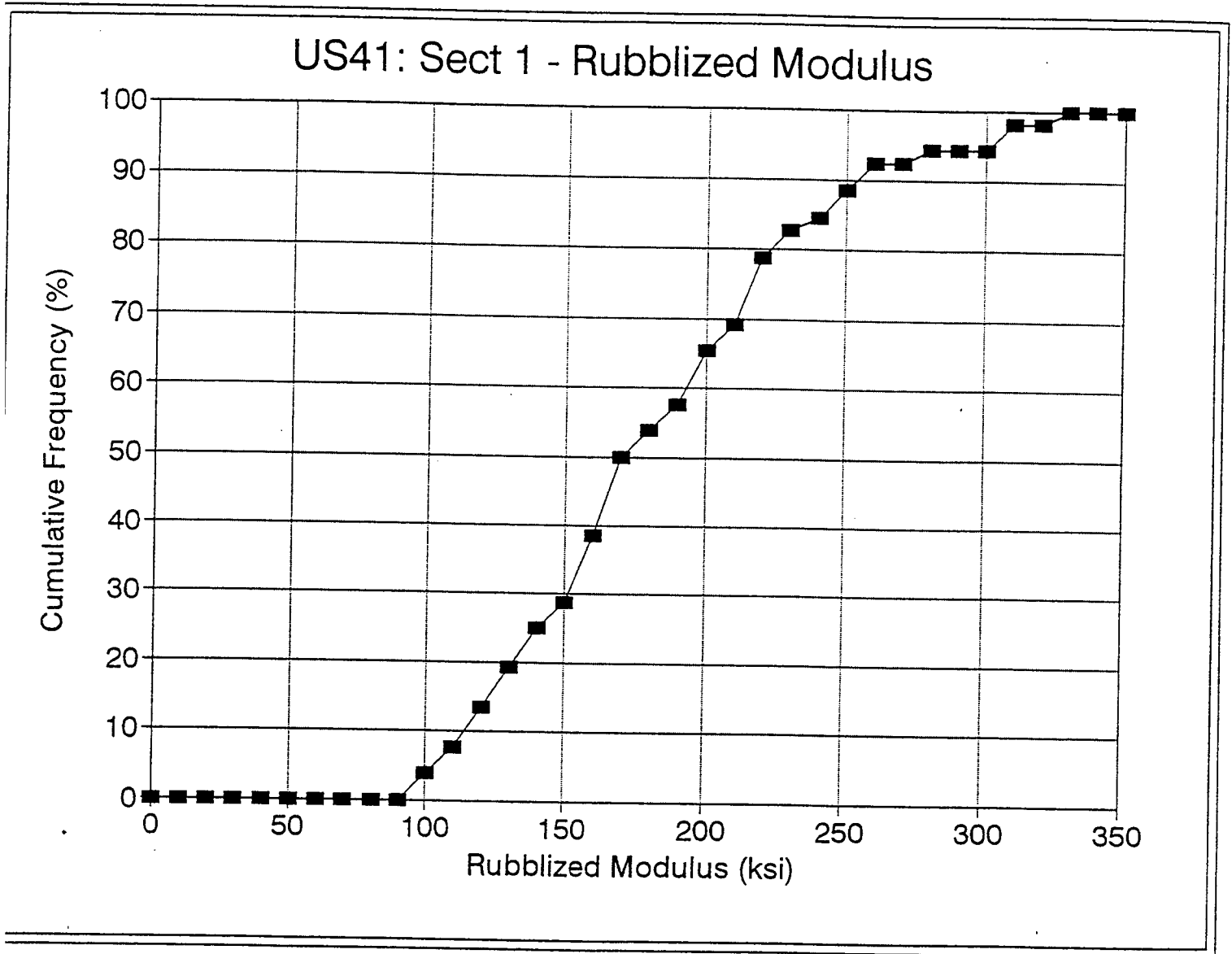
Figure 2. US41: Sect 1 - Rubblized Modulus
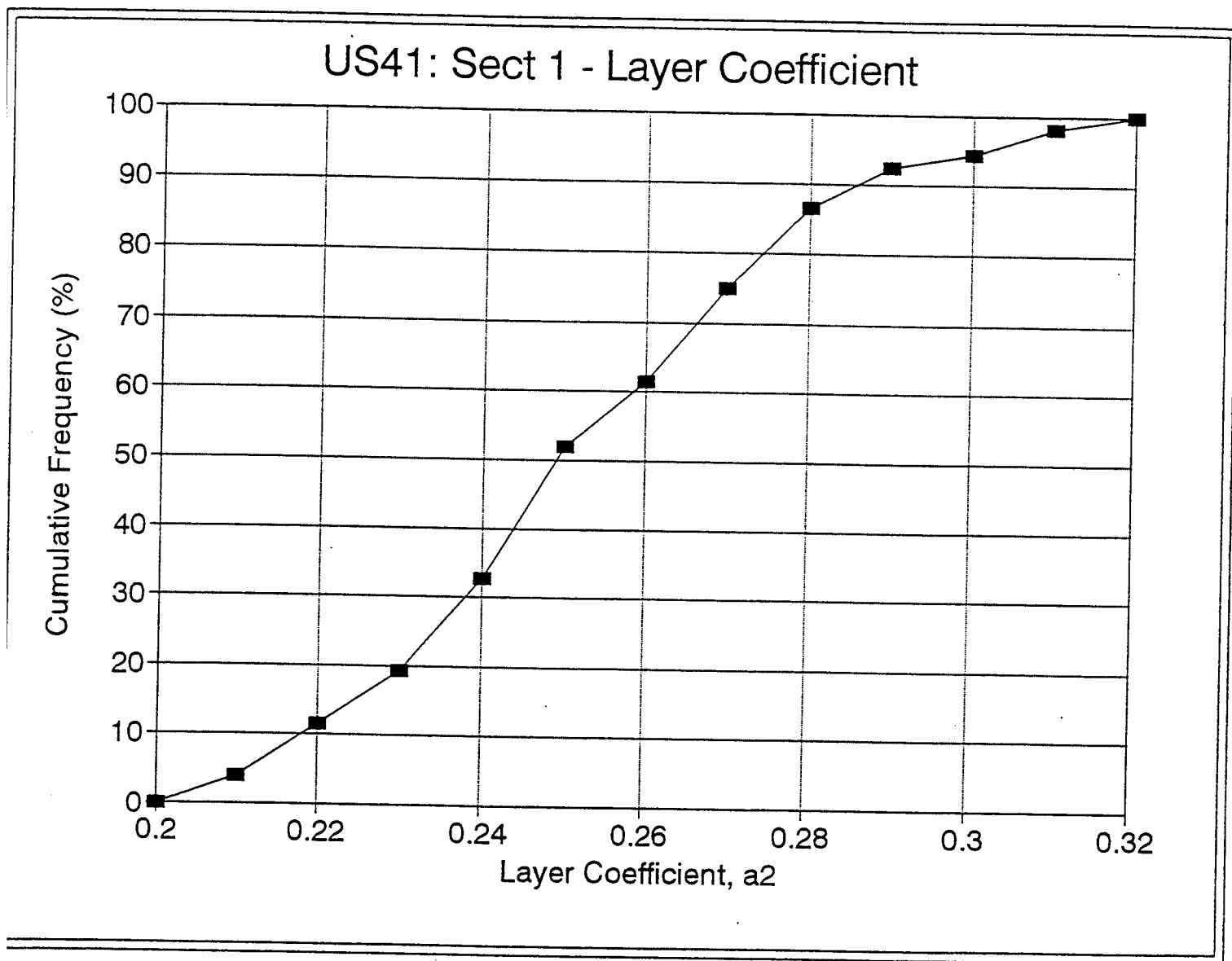
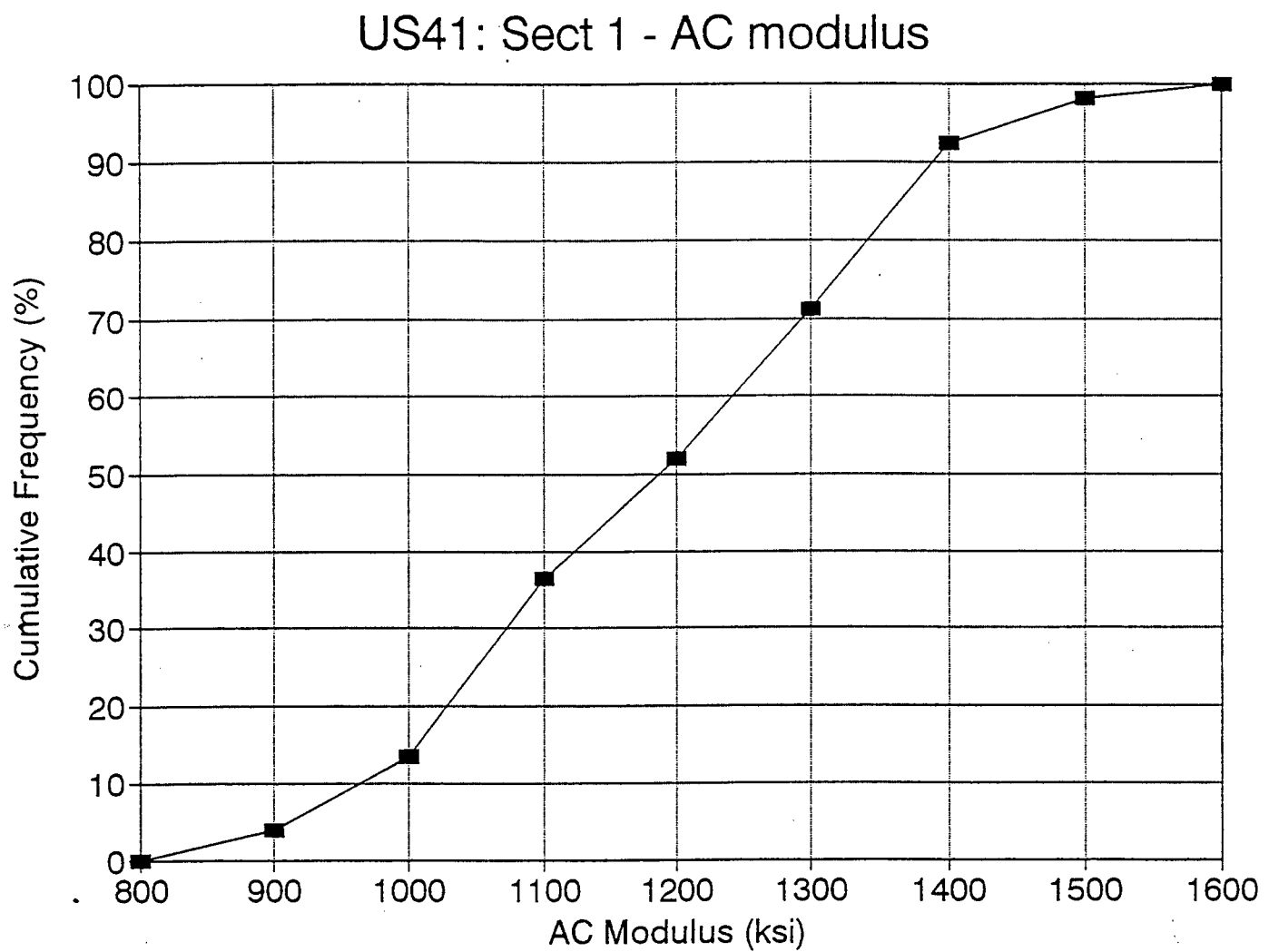Figure 3. US41: Sect 1 - Layer Coefficient
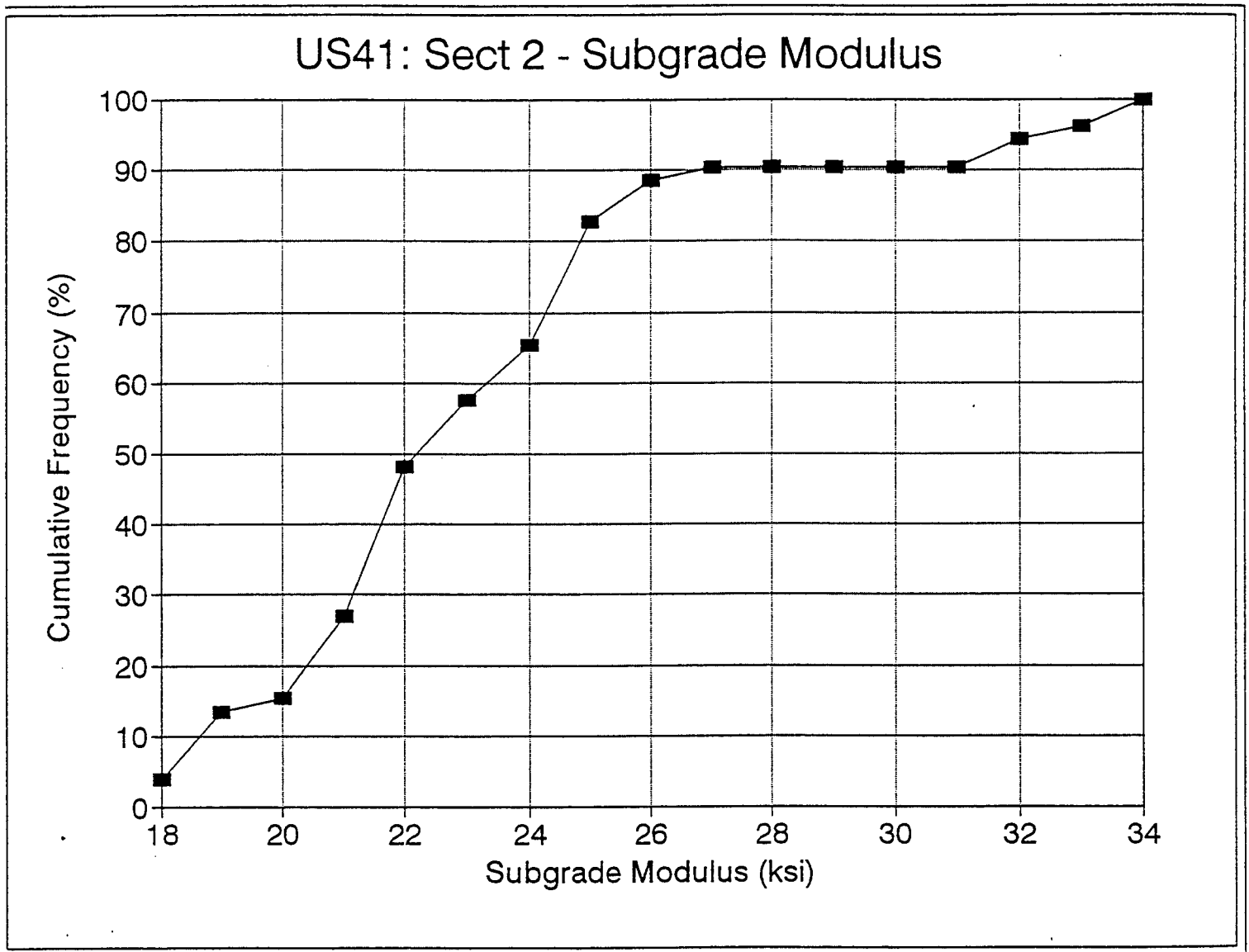
Figure 4. US41: Sect 1 - AC Modulus

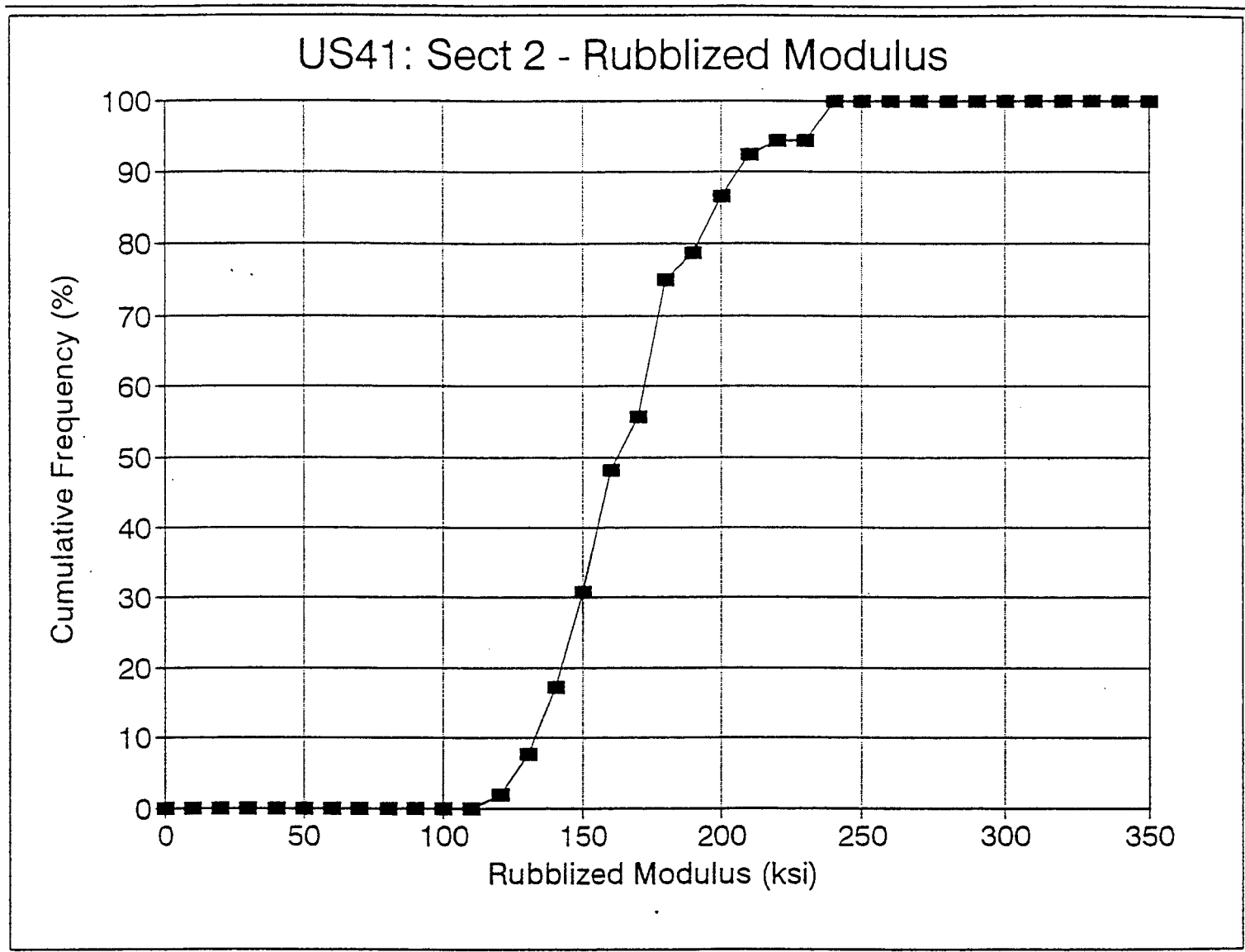Figure 5. US41: Sect 2 - Subgrade Modulus
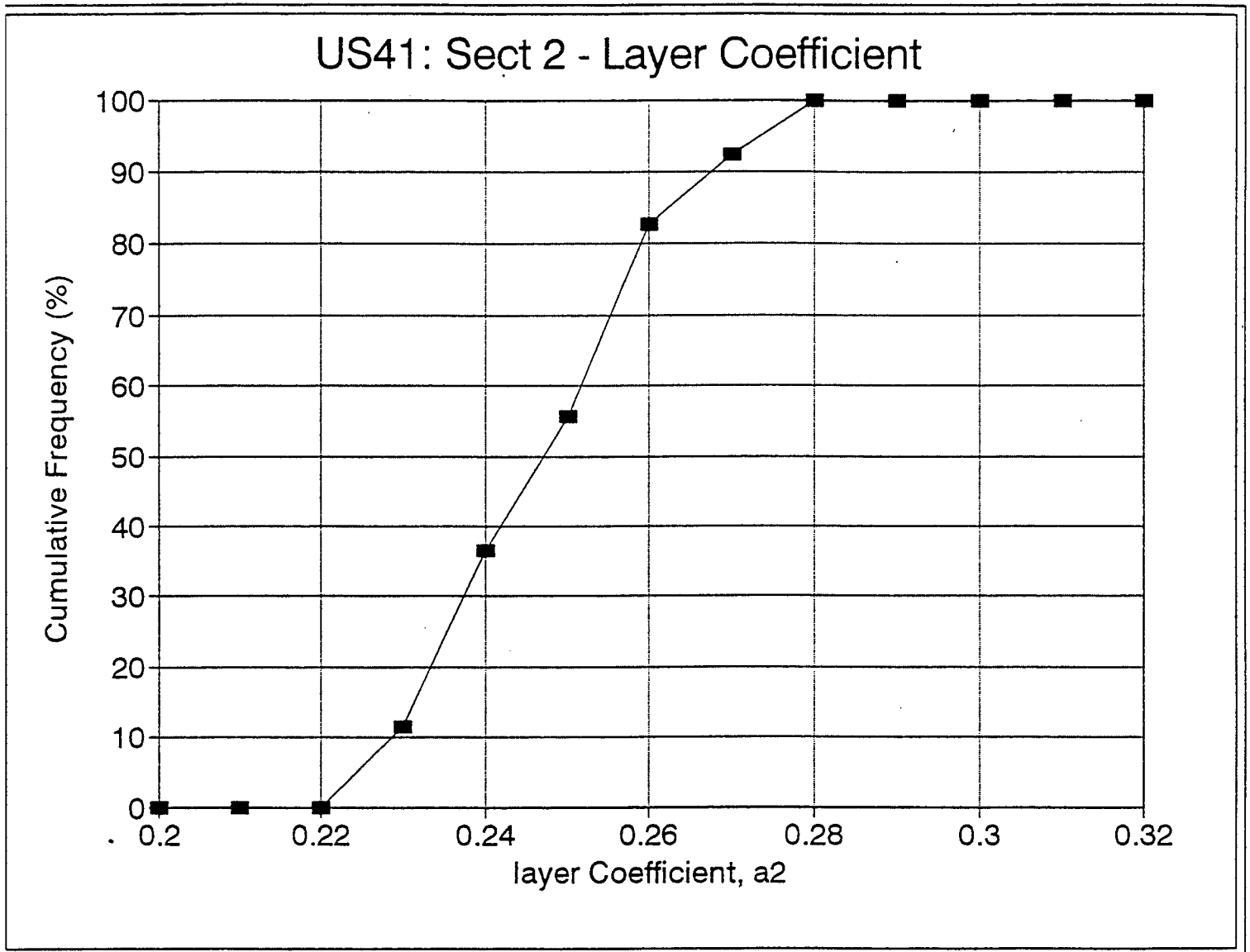
Figure 6. US41: Sect 2 - Rubblized Modulus

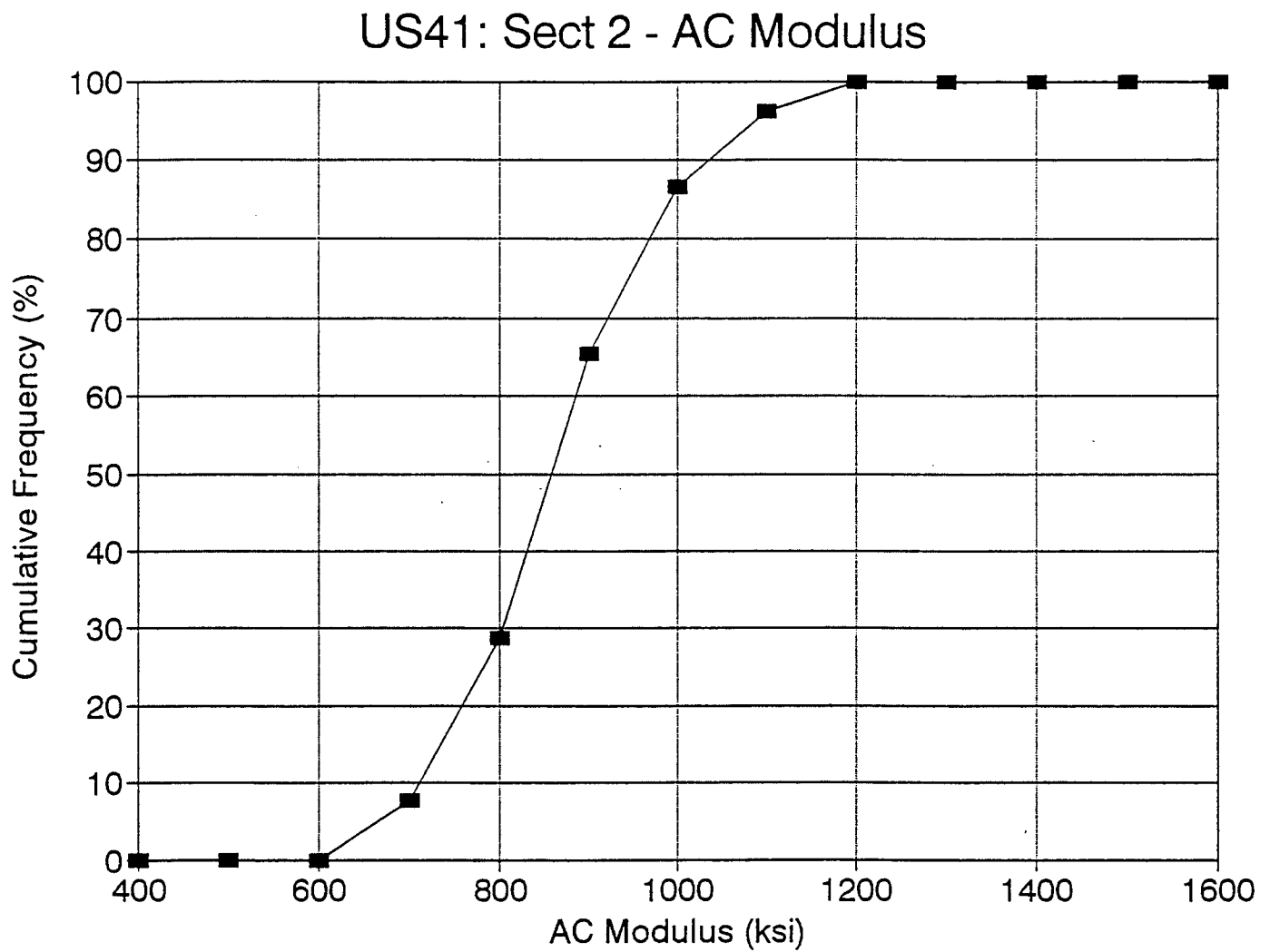Figure 7. US41: Sect 2 - Layer Coefficient

Figure 8. US41: Sect 2 - AC Modulus

Appendix A

Conversion Factors:

Multiply English Units by conversion factors provided to get equivalent Metric Units.
Divide Metric Units by conversion factors provided to get equivalent English Units.

Pa	=	1.4504×10^{-4}	lbf/in^2
kPa	=	0.14504	lbf/in^2
m	=	3.28083	ft
km	=	3280.83	ft